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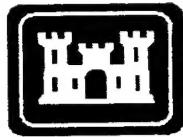
D R A F T

CUTOFF WALLS AND CAP FOR LIME AND M-1 SETTLING BASINS

**ROCKY MOUNTAIN ARSENAL
COLORADO**

NOVEMBER 1990

**Rocky Mountain Arsenal
Information Center
Commerce City, Colorado**



**US Army Corps
of Engineers**

Omaha District

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IMPLEMENTATION DOCUMENT
FOR

CUTOFF WALLS AND CAP FOR LIME AND M-1 SETTLING BASINS
ROCKY MOUNTAIN ARSENAL, COLORADO

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PART 1 - GENERAL DESCRIPTION

1. PURPOSE. The purpose of the project was to develop a design for the Interim Response Actions (IRA) at the Lime and M-1 Settling Basins at Rocky Mountain Arsenal (RMA), Commerce City, Colorado. The purpose of the IRA at the Lime and M-1 Settling Basins is to mitigate the threat of release from the Basins on an interim basis, pending determination of the final remedy in the Onpost Record of Decision (ROD). The IRA for the M-1 Basins also includes treatment of the waste materials in the basins with in-situ vitrification (ISV), which is being designed by contract with Woodward-Clyde Consultants.

2. AUTHORIZATION.

2.1 AUTHORITY. This project was authorized by DD Form 448 from Program Manager for Rocky Mountain Arsenal.

2.2 DECISION DOCUMENTS. The IRA for the Lime Settling Basins consists of relocation of sludge material, which has been deposited around the Lime Settling Basins, to the Lime Settling Basins area; construction of a 360-degree subsurface barrier around the basins; construction of a soil and vegetative cover over the material; and installation of a ground-water extraction system. The IRA for the M-1 Settling Basins consists of construction of a temporary 360-degree subsurface barrier such as a slurry wall or sheet pilings around the M-1 Settling Basins area, and the treatment of the waste materials in the basins with in-situ vitrification.

3. CRITERIA. Criteria used in the remedial design are referenced in Part 2 of this report, and are based on applicable local, state, and federal regulations.

4. PROJECT DESCRIPTION. The following is the project description as quoted from applicable portions of the Decision Documents for the Lime Settling Basins and M-1 Settling Basins at Rocky Mountain Arsenal.

4.1. SITE NAME, LOCATION AND DESCRIPTION.

4.1.1. LIME SETTLING BASINS. The Lime Settling Basins are located north of the South Plants area, just north of December 7th Avenue along the southern edge of the southwest quarter of section 36. The Lime Settling Basins occupy about 5 acres. For the purpose of the alternatives assessment, it was estimated that approximately 80,000 cubic yards of sludge within the basins, plus approximately 26,000 cubic yards of sludge that had been placed adjacent to the basins for drying, would be addressed by the IRA.

4.1.2. M-1 SETTLING BASINS. The M-1 Settling Basins are located in the South Plants area, just south of December 7th Avenue along the northern edge of the northwest quarter of Section 1. The basins and the berms surrounding them, all of which are now buried and partially built upon, occupy an area of approximately 34,500 square feet. For the purpose of the alternatives assessment it was estimated that approximately 6,400 cubic yards of sludge plus 2,600 cubic yards of overburden would be addressed by the IRA.

4.2. SITE HISTORY.

4.2.1. LIME SETTLING BASINS.

4.2.1.1. During the 1940's and 1950's, wastewater from the production of Army agents was routinely treated prior to discharge to unlined evaporation ponds. This treatment involved the addition of lime to the wastewater to precipitate metals and reduce the arsenic concentration. Wastewaters produced in the South Plants were channeled through the Lime Settling Basins prior to gravity discharge to Basin A, just to the north. The precipitation process produced a lime sludge that contained elevated levels of heavy metals, arsenic, and mercury. Subsequent discharges of pesticide production wastewater resulted in the addition of pesticide to the Lime Settling Basins sludge. The Lime Settling Basins were taken out of service in 1957.

4.2.1.2. A number of studies have been completed to characterize the nature and extent of contamination in the soil, sludge, and ground-water in the vicinity of the Lime Settling Basins. These studies are referenced in the Decision Document and the results are consistent with the site history. The soil and sludge contain elevated levels of organochlorine pesticides, organosulfur compounds, arsenic, mercury, and ICP metals (cadmium, chromium, copper, lead, and zinc).

4.2.2. M-1 SETTLING BASINS.

4.2.2.1. The M-1 Settling Basins were constructed to treat fluids from the lewisite facility. Two basins were constructed in 1942, and a third was constructed in 1943 when the original two filled with solids. All three were unlined, and each measured approximately 90 feet wide, 115 feet long, and 7 feet deep. In addition to the waste fluids from the lewisite disposal facility, the basins may have contained lesser amounts of waste materials from alleged spills within the acetylene generation building, the thionyl chloride plant, and the arsenic trichloride plant, which may have been routed through floor drains and the connecting piping to the basins. The basins also received a considerable amount of mercury chloride catalyst, possibly from a spill.

4.2.2.2. The liquids discharged into the basins first passed through a set of reactor towers where calcium carbonate was added, then through a wood trough into the M-1 Settling Basins where the arsenic precipitated out of solution. The liquid from the settling basins was decanted through an 18 inch diameter pipe to the Lime Settling Basins where final treatment occurred, before being routed to Basin A. The M-1 Settling Basins were backfilled, probably in 1947, and are now covered with soil. Portions of the basins are covered with structures. These structures will be relocated as part of this IRA before implementation of the ISV treatment.

4.2.2.3. Based on several investigations, the contaminants in the waste material in the M-1 Settling Basins are primarily arsenic (about 8 percent) and mercury (about 0.5 percent), with the bulk of the material being oxides or carbonated of calcium. Organochlorine pesticides and other organics have also been found in the near-surface soils. The bottoms of the basins appear to be about 7 feet below ground surface, based on as-built drawings and field investigations.

PART 2 - DESIGN REQUIREMENTS AND PROVISIONS

1. GEOLOGY.

1.1 GENERAL GEOLOGY.

1.1.1 Physiography. The Rocky Mountain Arsenal (RMA) lies within the Colorado Piedmont section of the Great Plains physiographic province. This area is characterized by surface deposits of wind-blown and alluvial materials. The Arsenal lies near the eastern edge of the valley along the South Platte River. The topography of the Rocky Mountain Arsenal area consists of gently rolling hills with occasional prominent hills which contain bedrock outcrops.

1.1.2 Description of Overburden. Overburden in the Rocky Mountain Arsenal area consists of both alluvial and eolian deposits of silts, clays, sands, gravels and a few cobbles.

1.1.2.1 There are several distinct deposits that make up the overburden that have been identified in the Rocky Mountain Arsenal area. The Quaternary units, from oldest to youngest, include the Verdos, Slocum, Louviers, Broadway, unnamed loess, unnamed eolian, Piney Creek, and Post Piney Creek. The older alluvium is primarily coarse-grained sands and gravels whereas the younger alluvium and the eolian deposits are primarily finer grained materials. The alluvial materials were deposited in irregular, braided channel environments creating typical lenticular deposits. The eolian materials are generally silts and fine sands. The thickness of these deposits in the vicinity of the Rocky Mountain Arsenal varies from 130 feet thick to less than 20 feet. These materials are generally unconsolidated and lie unconformably on the Cretaceous-Tertiary Denver Formation.

1.1.3 Bedrock Stratigraphy. The bedrock unit lying directly below the Quaternary alluvium is the Denver Formation. Immediately underlying the Denver Formation is the Arapahoe Formation. The thickness of the Denver Formation in the vicinity of the Rocky Mountain Arsenal varies from 200 to 500 feet. The Denver Formation was derived from the erosion of basaltic and aesthetic material and was deposited by fluvial processes. The Denver Formation consists of units of interbedded siltstones, claystones, sandstones and lignite. A low permeability volcaniclastic layer is present in the upper portion of the Denver Formation. This volcaniclastic layer contains lithic fragments and minerals in a bentonitic clay matrix which probably is the product of a weathered volcanic ash deposit. Sandstone layers of the Denver Formation are usually discontinuous, lense-shaped and generally grade laterally and vertically into shales and siltstones. The lignite layers are more continuous than the sandstone layers and are usually fractured.

1.1.4 Bedrock Structure. The Rocky Mountain Arsenal facility lies in the northwestern portion of the Denver Basin. The Denver Basin is an extensive, oval-shaped, structural depression extending from eastern Colorado and eastern Wyoming into western Kansas and western Nebraska. The sedimentary rocks that fill the Denver Basin are predominantly shales, sandstones, conglomerates and occasionally some limestones. The gently dipping slope of

shallow bedrock formations of the Denver Basin is one degree or less in the vicinity of the Rocky Mountain Arsenal and is predominantly to the southeast.

1.2 INVESTIGATIONS SUMMARY.

1.2.1 General. Pre-design investigations consisted of review of IR and IRA reports and field investigations of both the Lime Settling Basins and M-1 Basin areas. The field investigations included topographic surveys, drilling, and sampling, for geotechnical and chemical testing and in-situ permeability testing. Topographic surveys were conducted by Government personnel. Drilling, sampling and permeability testing was accomplished under contract with Woodward Clyde Consultants (WCC). Omaha District personnel visited the work site to oversee the work of the contractor. The discussion of the drilling, sampling, and permeability testing is a summary of the work done by WCC. A complete discussion of the field investigations performed by WCC is included in their report "Field Investigation, Lime and M-1 Settling Basins Slurry Walls, Rocky Mountain Arsenal, Commerce City, Colorado" dated September, 1990, Volumes I and II.

1.2.2 Topographic Surveys. In order to develop existing site conditions, a topographical survey was conducted to establish horizontal and vertical control. Subsequent mapping was prepared. The mapping, which contains all topographic features, was drawn at 1"=50' for the Lime Basins Area, and at 1"=20' for the M-1 area. A 1 foot contour interval was used. The survey was also used to determine the field locations of the Lime Settling and M-1 Basins. Since the M-1 Basins are buried, stakes were placed at the boundaries. The locations were determined from as-built drawings and reviewing aerial photography.

1.2.3 Exploration Drilling for Lime Settling Basins. Field investigations for the Lime Settling Basins were conducted by WCC during June and July 1990. Field investigations consisted of electro-magnetic surveys for locating buried metallic objects, exploratory drilling, slug tests for hydraulic conductivity analysis, ground-water sampling, soil sampling and analyses, and bulk sampling for compatibility testing and borrow area analysis. All drilling except the borrow investigations was conducted in level B protection.

1.2.3.1 Equipment and Personnel. Drilling was accomplished by Layne-Western Co. under contract to WCC. Drilling and sampling was accomplished by drilling with 6 1/4-inch OD hollow stem augers using a CME 75 or CME 750 drilling rig. The majority of samples were obtained by a 3-inch OD, stainless steel split spoon driven by a 140-pound hammer. Continuous auger cores were taken of bedrock in polybutyrate tubes. Logging and sampling of borings were done by WCC personnel.

1.2.3.2 Boring Locations and Purpose. A total of 30 borings were drilled for the investigation of the Lime Settling Basins Project. Nineteen borings were drilled along the alignment of the slurry trench to identify the subsurface materials and to determine the consistency, density and moisture content of the overburden, and the nature and characteristics of bedrock including degree of weathering, fracturing, and cementation, and relative hardness. Eight borings were drilled outside the slurry trench area to determine

the extent of waste material that had been removed from the Lime Settling Basins. Three wells were installed inside the slurry trench area for slug tests to determine the hydraulic conductivity of the overburden aquifer. Samples were taken from all borings for geotechnical analyses; compatibility testing; and chemical analyses.

1.2.3.3 Slug Tests. Slug tests were conducted in wells installed in borings LSB-15, LSB-34, and LSB-35 on 24 and 25 July 1990. These slug tests were conducted to ascertain the hydraulic conductivity of the overburden aquifer within the isolation cell, particularly for the design of the ground-water extraction system. The wells were constructed of 4-inch ID PVC riser pipe and 10 feet of slotted PVC casing used for the well screen. The bottom of the screens were placed at the top of bedrock. The screens were sand packed to the top of the aquifer then sealed with hydrated bentonite pellets and bentonite grout to the ground surface. The wells were developed with a 3-inch diameter steel surge block. After development the wells were allowed to recover two weeks before initiation of the slug tests. Slug tests were conducted using an automated data logger, 10-psi range pressure transducer probe, and a 5-foot long slug constructed of PCV pipe filled with sand and capped at each end. A falling head slug test and a rising head slug test were conducted in each well. The tests were continued until 90 percent of the induced head change was dissipated.

1.2.3.4 Backfilling Holes. All holes were backfilled with grout after completion.

1.2.4 Exploration Drilling for M-1 Basins. Field investigations were also conducted in July 1990. With the exception of slug tests, field investigations were identical to those conducted at the Lime Settling Basins.

1.2.4.1 Equipment and Personnel. Equipment and personnel involved in field investigations for the M-1 Basins were essentially the same as those at the Lime Settling Basins.

1.2.4.2 Boring Locations and Purpose. A total of 29 borings were drilled for the investigation of the M-1 Basins. Of these 12 were drilled for the design of the sheet pile wall, 2 were drilled for background geological information, and 15 were drilled within the basins to obtain data for the design of the in-situ soil vitrification project, primarily to characterize the waste sludge. As for the Lime Settling basins borings, borings drilled along the alignment of the sheet pile wall were drilled to determine the nature and character of the overburden and bedrock materials.

1.2.4.3 Backfilling Holes. All exploratory borings were backfilled with grout after completion.

1.3 SITE GEOLOGY.

1.3.1 Lime Settling Basins.

1.3.1.1 Bedrock. Bedrock beneath the Lime Settling Basins area is the Cretaceous-Tertiary Denver Formation. The unconformable contact between the bedrock and the overlying surficial deposits is irregular due to

erosion of the surface of the bedrock prior to the deposition of the surficial material. The uppermost portions of the Denver Formation are weathered and often fractured.

1.3.1.1.1 Lithology. The Denver Formation in the vicinity of the Lime Settling Basins consists of claystone and sandstone. The claystone is generally soft to moderately hard, brown, and blocky and is occasionally silty. Sandstone lenses are also frequently encountered. The sandstone units are fine-grained and vary from soft to hard, depending upon the degree of cementation and weathering, and fine grained. These sandstones also contain silt, thus making them less pervious. A thick, up to 15 feet, fine-grained sandstone lense occurs in the northern section of the isolation cell.

1.3.1.1.2 Bedrock Topography and Structure. The Denver Formation bedrock lies at depths of 13.5 to 27.5 feet below the surface in the Lime Settling Basins area. The local slope of the surface of the bedrock is very gentle, about two degrees, to the north-northeast. It also displays paleochannel valleys and benches. This type of paleotopography is due to stream erosion. The dip of the Denver Formation has not been determined, but it is probably the same as the regional dip, about one degree or less to the southeast.

1.3.1.2 Overburden. The overburden in the Lime Settling Basins area is of Quaternary age and is the result of deposition by the ancient Platte River drainage network and eolian processes.

1.3.1.2.1 Lithology. The thickness of the overburden ranges between 13.5 and 27.5 feet in the Lime Settling Basins area. The soils consist mostly of poorly graded, silty, fine-grained sand with moderate amounts of sandy, silty clay and minor amounts of clayey sand, sandy clay, silty clay, and clay. The sand ranges from loose to dense and the clay ranges from soft to very stiff. The overburden soil ranges from dry to saturated with moisture content increasing with depth.

1.3.1.2.2 Alluvial Aquifer Material. The aquifer material is generally unconsolidated, fine-grained sand or silty, fine-grained sand, and clayey fine-grained sand overlying the top of bedrock. The saturated thickness ranges from 9.5 to 21.0 feet.

1.3.2 M-1 Basins.

1.3.2.1 Bedrock. The bedrock beneath the M-1 Settling Basins area is the Cretaceous-Tertiary Denver Formation. The unconformable contact between the bedrock and the overlying surficial deposits is irregular due to erosion of bedrock prior to the deposition of the surficial material. The uppermost portions of bedrock are weathered and often fractured.

1.3.2.1.1 Lithology. The upper portion of the bedrock is weathered, soft to moderately hard, dark brown claystone occasionally interbedded with moderately hard to hard, fine-grained sandstone and sandy siltstone.

1.3.2.1.2 Bedrock Topography and Structure. The Denver Formation bedrock is located at depths of 9.0 to 14.5 feet below the surface in

the M-1 Settling Basins area. The slope of the surface of the bedrock is very gentle, less than one degree to the north. The bedrock surface was shaped by stream erosion. As at the Lime Settling Basins, the dip of the Denver Formation probably coincides with the regional dip of one degree or less to the southeast.

1.3.2.2 Overburden. The overburden material in the M-1 Settling Basins area is of Quaternary age and is the result of deposition by the ancient Platte River drainage network and eolian processes.

1.3.2.2.1 Lithology. The overburden in the M-1 Basins area is 9.0 to 14.5 feet in thick. The material consists mostly of unconsolidated, fine-grained, yellowish to grayish brown sand and silty sand with silt and clay of alluvial or eolian origin; surficial fill material; and chemical waste sludge. The fill extends from the ground surface downward, ranges from 2 to 11 feet thick and consists mostly of a mixture of clay, sand, and gravel occasionally mixed with sandstone and claystone. The chemical waste sludge ranges from 3.0 to 6.5 feet thick. Overburden ranges from dry to saturated with moisture content increasing with depth.

1.3.2.2.2 Aquifer Material. The aquifer material in the M-1 Basins consists of alluvial and eolian materials which are unconsolidated, poorly graded, fine grained sand and occasional silt. Saturated thickness ranges from 3.0 to 4.5 feet.

1.4 HYDROLOGY.

1.4.1 Regional Hydrology. The Rocky Mountain Arsenal lies within the drainage basin of the South Platte River. The South Platte River is approximately 3 miles west and northwest of the Arsenal. Ground-water flow in the Arsenal area is from southeast to northwest eventually discharging into the South Platte River. Ground water in the overburden is generally unconfined while ground-water in the bedrock units is confined. Ground water is unconfined where permeable bedrock units are exposed at the surface or in contact with the overburden. The aquifer units of greatest concern in the Rocky Mountain Arsenal vicinity are in the surficial Quaternary overburden deposits and permeable sandstones of the underlying Denver Formation. The bottom portion of the Denver Formation is a "buffer zone" of shale. This buffer zone is approximately 75 to 200 feet thick and acts as an aquitard between the Denver Formation and the lower bedrock formations of Arapahoe, Laramie Formation, Fox Hills Sandstone and the Pierre Shale.

1.4.1.1 Bedrock. Confined aquifers in the Denver Formation exist in the more permeable sandstones and lignite beds. These beds are separated from the overlying alluvial aquifer by shale or claystone. The Arapahoe Formation underlies the Arsenal area at a depth of 200 to 500 feet below the ground surface. Due to high-volume ground water withdrawals from the Arapahoe Formation over the past 100 years, the vertical gradient between the Denver and Arapahoe Formations in the vicinity of the Rocky Mountain Arsenal has changed from upward to downward. Recharge of the bedrock aquifers occurs from vertical leakage from the alluvial aquifers.

1.4.1.2 Overburden. Unconfined ground-water occurs in unconsolidated surficial alluvium or eolian deposits of sand. Ground-water flow in the alluvium is most rapid through coarse materials found in paleochannels, however, flow does occur throughout the saturated thickness of the overburden deposits. Thick, saturated alluvial deposits are capable of yielding large volumes of water.

1.4.2 Site Hydrology.

1.4.2.1 Lime Settling Basins.

1.4.2.1.1 Bedrock. The Denver Formation is saturated below the Lime Settling Basins and contains some confined aquifers. The most conductive units are generally subhorizontal layers of sandstones and siltstones confined by less permeable claystones. The ground-water flow in the bedrock aquifers is believed to be due north.

1.4.2.1.1.1 Hydraulic Analysis. Hydraulic conductivities for the Denver Formation aquifers vary both vertically and horizontally based on lithology and the degree of weathering and fracturing. Shales and claystones have a reported hydraulic conductivity ranging from 3.53×10^{-6} to 3.53×10^{-8} cm/sec. Unfractured claystones may be as low as 3.53×10^{-12} cm/sec (Stollar and Assoc. 1989). A conservative value of 1.0×10^{-8} cm/sec for the vertical hydraulic conductivity for the claystone was used in calculations for this project. The hydraulic conductivities of the various sandstones have been reported to range from 1.06×10^{-5} to 1.41×10^{-3} cm/sec (Stollar and Assoc. 1989).

1.4.2.1.2 Overburden. The Lime Settling Basins are situated in a local topographic low area. The Lime Settling Basins are hydrogeologically downgradient from the M-1 Settling Basins and the South Plants area. The ground-water flow direction is about due north with a gradient of 0.023 ft/ft. There is ponded water inside the lime settling basins and it has been determined that the ponded water reflects the water table.

1.4.2.1.2.1 Hydraulic Analysis. Slug tests were conducted to determine the hydraulic conductivity for the fine-grained materials in the overburden at the Lime Settling Basins. Three slug tests were conducted, with one test conducted near the center of the isolation cell and the other two conducted 100 feet south of the north wall of the isolation cell. The hydraulic conductivity near the center of the isolation cell was determined to be 2.0×10^{-3} cm/sec. The two tests 100 feet south of the north wall indicated hydraulic conductivities of 1.0×10^{-4} cm/sec and 2.0×10^{-4} cm/sec. In order to perform conservative analyses for the design of the project the lowest hydraulic conductivity was used in all calculations. Review of boring logs and mechanical analyses of samples obtained during the exploration program support the use of the smallest value obtained from the slug tests.

1.4.2.2 M-1 Basins.

1.4.2.2.1 Bedrock. The Denver Formation is saturated below the M-1 Basins and may contain some confined aquifers. Subhorizontal layers of sandstones and siltstones, confined by less permeable claystones, are generally

the most permeable units of the Denver Formation. Ground-water flow within these aquifer units is generally due north.

1.4.2.2.1.1 Hydraulic Analysis. The aquifers within the Denver Formation have hydraulic conductivities that vary both vertically and horizontally based on lithology. The claystones and shales have reported hydraulic conductivities ranging from 3.53×10^{-6} to 3.53×10^{-8} cm/sec. The sandstones have a reported hydraulic conductivity ranging from 1.06×10^{-5} to 1.41×10^{-3} cm/sec. The uppermost unit of the Denver Formation below the M-1 Basins is claystone.

1.4.2.2.2 Overburden. Paleotopographic influences and localized mounding of ground-water direct the flow of ground-water in the M-1 Basins area due north to slightly northwest. The ground-water gradient ranges from 0.008 to 0.011 ft/ft. The water table varies seasonally between 5 and 10 feet below ground surface. Current water levels range from 7.8 to 10.0 feet below ground surface. The thickness of the saturated zone ranges from about 3.0 to 4.5 feet.

1.4.2.2.2.1 Hydraulic Analysis. The hydraulic conductivities for the overburden aquifer has been reported as ranging from 6.0×10^{-3} cm/sec to 2.4×10^{-3} cm/sec.

1.5 CONTAMINATION.

1.5.1 Lime Settling Basins.

1.5.1.1 Soils. Soil contamination at the Lime Settling Basins has been investigated previously. Contaminants detected have included raw materials, such as mustard agent-related compounds, manufacturing by-products, such as volatile aromatic solvents, and degradation products from the synthesis of insecticides. Previous borings indicate detectable concentrations of organochlorine pesticides (OCP's). The following OCP's were detected: dieldrin, with concentrations from 0.6 to 70 $\mu\text{g/g}$, aldrin, with concentrations up to 600 $\mu\text{g/g}$, endrin, with concentrations up to 200 $\mu\text{g/g}$, and isodrin, with concentrations up to 300 $\mu\text{g/g}$. Other contaminants found were organosulphur compounds of chlorophenylmethyl sulfide, chlorophenylmethyl sulfoxide, and chlorophenylmethyl sulfone with concentrations up to 50 $\mu\text{g/g}$. DDT was also found in an isolated area with a concentration of 7 $\mu\text{g/g}$. Volatile organic compounds (VOC's) were detected in some of the previous deeper borings. Chloroform was detected at concentrations ranging from 2 to 7 $\mu\text{g/g}$. Benzene was detected at concentrations ranging from 5 to 6 $\mu\text{g/g}$ and chlorobenzene was detected at a concentration of 2 $\mu\text{g/g}$. The most prevalent metals were arsenic and mercury. Arsenic concentrations were detected as high as 370 $\mu\text{g/g}$. Mercury was detected at concentrations of 0.159 $\mu\text{g/g}$. Elevated concentrations of copper, lead, zinc, cadmium, and chromium were also detected. Tetrachloroethene was also detected at a concentration of 0.25 $\mu\text{g/g}$.

1.5.1.2 Ground Water. Ground-water contaminants in the alluvial aquifer have been detected at the Lime Settling Basins site during previous investigations. These contaminants include VOC's (volatile organic compounds), aromatics, metals, and OCP's (organochlorine pesticides). High concentrations

of various VOC's were detected. Arsenic, mercury, chromium, and copper were metals that were detected from previous monitoring well samples.

1.5.2 M-1 Basins.

1.5.2.1 Soils. High concentrations of arsenic and mercury were found in and around the M-1 Basins from depths of 0.5 to 7.0 feet. The concentrations within the basins ranged from 0.01% to 11%. These concentrations generally decreased below the 7-foot depth. Dieldrin, DCPP and BCHPD have also been reported in significant concentrations.

1.5.2.2 Ground water. Previous investigations indicate high concentrations of arsenic and mercury are present in the ground water downgradient of the M-1 Settling Basins.

2. SLURRY TRENCH CUTOFF WALL (LIME SETTLING BASINS).

2.1 Criteria. The Decision Document for the Lime Settling Basins requires the containment system consist of a 360 degree subsurface barrier, vegetative cover, ground-water removal system to maintain a negative head in the isolation cell, evaluation of ground-water diversion, and evaluation of potential contamination of bedrock aquifers.

2.2 References. The following references were used during the design process:

EPA-540/2-84-001, Slurry Trench Construction for Pollution Migration Control.

EPA 600/2-87-063, Investigation of Slurry Wall Design and Construction Methods for Containment of Hazardous Waste.

Millet and Perez, "Current USA Practice: Slurry Wall Specifications", ASCE Journal of Geotechnical Engineering, August 1981.

D'Appolonia, "Soil-Bentonite Slurry Trench Cutoffs", ASCE Journal of Geotechnical Engineering, April 1980.

ASTM STP 874, Hydraulic Barriers in Soil and Rock, 1985.

Xanthakos, Slurry Walls, 1979.

Zappi, Shafer, and Adrian, "Compatibility of Soil- Bentonite Slurry Wall Backfill Mixtures With Contaminated Ground-water", Proceedings of Superfund '89 Conference, HMCRI, November 27-29, 1989.

Schulze, Barvenik, and Ayres, "Design of Soil-Bentonite Backfill Mix for the First Environmental Protection Agency Superfund Cutoff Wall".

API RP 13B-1, API Recommended Practice: Standard Procedure for Testing Drilling Fluids.

API Spec. 13A, API Specification for Oil Well Drilling Fluid Materials.

Plans and Specifications, Helen Kramer Landfill Superfund Site, Mantua Township, New Jersey, prepared by the Kansas City District

Plans and Specifications, Kane and Lombard Superfund Site, East Baltimore, Maryland, prepared by EA Engineering Science and Technology Inc.

2.3 Compatibility Study. The presence of chemical contaminants in soil and/or ground-water may significantly alter the rate of water movement through

a soil medium. The purpose of compatibility testing is to find the mixture of backfill soil, bentonite, and tap water which will produce the lowest permeability of contaminated ground water over time. The Missouri River Division (MRD) Laboratory in Omaha will perform the compatibility studies. On 1 October 1990, MRD lab personnel proposed changes in the testing procedure based on preliminary results of a similar compatibility study presently being conducted for the Forest Waste Superfund Site. The updated test request (dated 10 October) reflects the changes. The updated procedure is described here. After selection of a bentonite source using the free swell and filter cake compatibility tests, optimization testing (one or two-day fixed wall permeability tests) determines the most economical combination of bentonite, backfill soil, and water which yields a permeability of 1×10^{-7} cm/sec or less. That combination is used in permeameter tests utilizing both contaminated ground-water and tap water.

2.3.1 Bentonite Sources. Several drillers supply companies in Colorado and two out-of-state companies were contacted to identify potential bentonite suppliers and to obtain samples for conducting compatibility studies. Samples from the following companies were sent to the MRD Laboratory:

Golden Drilling Fluids and Supplies Inc.
Golden, CO
Regular

Dean Bennett Supply Company
Denver, CO
Mudgel

H & H Bentonite and Mud Inc.
Grand Junction, CO
BH-Natural and AS 85

Black Hills Bentonite Company
Palatane, IL
S-5 Natural

Wyo-Ben Inc.
Billings, MT
Hydrogel

The free swell test (EPA Report Number PB 87-229688) and filter cake compatibility test will be used to select the bentonite for this project. Two free swell tests will be performed for each bentonite sample; one using contaminated ground-water and one using RMA tap water (see following paragraph). The bentonite which exhibits the least amount of variation between the tap water and contaminated ground-water test will be selected for the backfill mixtures.

2.3.1.1 Natural vs. Treated Bentonite. The Corps slurry trench guide spec (several years old) specifies natural bentonite only. However, many slurry trench references (EPA-540/2-84-001, Millet and Perez, D'Appolonia, Xanthakos) say that practically all commercially available bentonite contains chemical additives; it is more a matter of how much is added. A memo from Geo-Con Inc., the Kane and Lombard contractor states that most slurry trench

specifications allow treated bentonites which conform to API Spec 13A Section 4 to be used. On the Kane and Lombard project only natural bentonite which conforms to API Spec 13A Section 5 (a new section) was allowed. Geo-Con experienced some problems during construction that they attribute to the natural bentonite. He recommends not using API 13A Section 5 natural bentonites for slurry trenches. Due to time constraints in both this project and the Forest Waste project it was decided to stick with the Corps spec and use only natural bentonites. In the future when time becomes available the MRD lab may do some comparisons between natural and treated bentonites to address this question. Of the bentonites sent to the MRD lab, only 2 (H & H Bentonite's BH-Natural and Black Hill's S-5 Natural are API 13A Section 5 natural and only those two will be used for testing.

2.3.2 Water Samples. To simulate field conditions at the Arsenal site, samples of tap water and contaminated ground water were collected during the pre-design field investigations. Tap water will be used to mix the slurry and backfill materials and ground water will be used as a permanent.

2.3.3 Backfill Soil Samples. Two backfill soils will be tested; the soil to be excavated from the trench and soil from an uncontaminated borrow area on the arsenal. Soil from the trench alignment has been collected as part of the pre-design field investigations. After screening for Army agents, samples from the borings located on the northern half of the slurry trench cell were composited and tested for grain size distribution, Atterberg limits, and water content at the Arsenal. This soil was sent to the MRD Lab for compatibility studies. Prior to the pre-design field investigations it was decided to use soil from the northern boundaries of the trench for compatibility studies. This is because the groundwater flow in the area is toward the north and the highest levels of contamination found in the previous studies is to the north of the Lime Basins; therefore that soil should provide the "worst case" testing condition. The boring logs along the trench alignment are very consistent: fill or sludge overlying SM (USCS classification), overlying CL-CH, overlying claystone. The mechanical analyses of the composited samples are also very similar, containing between 17 and 40 percent fines. Due to the overall consistency and the desire to use mostly soil from the northern boundaries while also assuring representative samples, it was decided to thoroughly mix samples from the following borings into one composite: 9, 10, 17, 22, 23, 24, 25 and 26. This represent all borings along the northwest, north, and northeast boundaries of the isolation cell.

2.3.3.1 Borrow Materials. Corps personnel decided to use a clay borrow area used in the remediation of Basin F. Four test pits were excavated in this area. Approximately 150 pounds of soil was collected and sent to the MRD Lab. As there is a limited amount of this material available, this soil will be used as a possible source of fines only and not the primary borrow material. This material classifies as CL, with a liquid limit of 34.6, plastic limit of 13.5, plasticity index of 21.1, and 69.9% finer than the No. 200 sieve. Stockpiles of soil excavated from the spillway construction at the Lower Derby Dam on the Arsenal will be used as the primary alternate borrow. Samples of this material were brought to the MRD lab on August 31, 1990 and will be tested for grain size distribution, Atterberg limits and moisture content prior to optimization testing.

2.3.3.2 Chemical Screening of Borrow. Prior to compatibility studies, both borrow soils will be tested for TCLP (Toxicity Characteristics Leaching Procedure), TOC (Total Organic Carbon), sodium, calcium, magnesium, potassium, pH, and cation exchange capacity.

2.3.4 Sample Preparation. The backfill soil samples will be oven-dried at 65 degrees Celsius for 2 to 4 days. The soils will then be broken up, thoroughly mixed, and passed through a U.S. Standard Sieve No. 4. The RMA tap water shall be added to the dried and mixed samples until the original field water content is reached. These reconditioned composite samples shall then be stored for 3 to 6 days in sealed containers at room temperature. During this storage period the samples will be mixed daily.

2.3.4.1 The bentonite slurry shall be prepared by mixing the RMA tap water with the previously selected bentonite source. The slurry shall be mixed with enough water to pass through a Marsh funnel in 40 to 50 seconds. The slurry shall be tested for density, water content, pH, viscosity, and fluid loss.

2.3.5 Optimization Testing. Short-term (1 or 2 days) permeability tests will be performed to determine the most economical combination of bentonite, water, fines and coarse grained soil with a permeability of 1×10^{-7} cm/sec or less. Since tap water and backfill soil are available on the Arsenal, it is anticipated bentonite will be the highest cost item.

2.3.5.1 Three samples (two specimens each) of the insitu slurry trench soil will be prepared containing 0, 2, and 4 percent dry bentonite by weight. Bentonite slurry with a viscosity of 40 seconds (Marsh funnel) will be added to each sample to obtain an approximate 5 inch slump. Fixed wall permeability tests will be run on the 6 specimens. "Total Percent Bentonite vs. Permeability" will be plotted. If permeability values are not less than or equal to $1 \times 10^{EE-7}$ cm/sec, the above procedure will be repeated with the addition of enough clay borrow soil to make the fines content approximately 10 percent higher than the original insitu composite. If those permeability values are not less than or equal to $1 \times 10^{EE-7}$ cm/sec, the procedure will be repeated with the addition of clay borrow soil to make the fines content approximately 20 percent higher than the original insitu composite. If permeability values are still too high, the procedure will be repeated with the addition of clay borrow to make the fines content approximately 30 percent higher than the original insitu composite.

2.3.5.2 The optimization testing procedure will also be performed using the random fill borrow soil in place of the insitu soil. Due to the presence of contaminants in the insitu soil there is a possibility that none of the mixtures of insitu soil, fines and bentonite will produce a permeability on the order of magnitude of 10^{EE-7} cm/sec. If this happens and a mixture of random fill, fines, and bentonite produces an acceptable permeability then only random fill borrow will be used for construction, and long-term permeability tests will be performed using only random fill borrow as the principal soil constituent. If the desired permeability is obtained by mixtures including both insitu soil and random fill borrow then long-term permeability tests will be performed using

both principal soil constituents, and the decision of which soil to use for construction will be made based on the results of those tests.

2.3.6 Permeameter Sample Preparation. Samples for long-term permeability tests will be prepared according to subparagraph 2.3.4, Sample Preparation. The backfill mixtures shall be stored in sealed containers at room temperature until loading into the permeameters for permeability testing. Atterberg limits, fines content, porosity, density, water content, specific gravity, cation exchange capacity, and pH of the backfill mixtures will be determined. Before the backfill materials are loaded into the permeameters, comments on the general appearance, i.e. color and texture of the material before permeameter testing shall be recorded. The backfill materials and bentonite slurry shall be photographed.

2.3.7 Permeameter Testing. Permeameter testing will be conducted in accordance with the Army Corps of Engineers Manual EM 1110-2-1906 using back pressure saturation and downflow conditions. Flexible wall permeameters shall be loaded with each of the backfill mixtures and leached with RMA tap water until one porewater volume has passed through the backfill mixtures. A total of three permeameter tests shall be run on each backfill mixture. One of the three tests for each backfill mixture will serve as a control test. Control cells will be leached with only RMA tap water throughout the duration of the test, and will consist of the selected mix with the percent bentonite which produced a permeability near $1 \times 10^{EE-7}$ cm/sec during optimization testing. The remaining two permeameters for each backfill mixture shall be leached with the contaminated ground-water, after one pore volume of RMA tap water has passed.

At least two pore volumes of contaminated ground-water will be leached through the backfill mixtures. One of these permeameters will contain the same mix and percent bentonite as the control cell. The other permeameter will contain the same mix as the control cell with a higher bentonite content that produced a permeability close to $1 \times 10^{EE-8}$ cm/sec during optimization testing. The samples will be compressed into the cell manually in order to reduce the amount of entrapped air.

2.3.7.1 Elevated hydraulic gradients shall be used in order to complete permeameter testing within a reasonable period of time (minimum two months). A pressurized nitrogen source will be used to achieve the required hydraulic gradients. The hydraulic gradient to be applied is 28. The confining pressure to be applied is 5 psi.

2.3.7.2 The permeameter influent will be tested for TOC, specific conductivity, bromide, pH, alkalinity, sodium, calcium, chloride, VOA (Volatile Organics), and BNA (Base Neutral Acid Extractible Organics) immediately prior to permeameter testing. Effluent from the permeameters will be collected and tested for the same chemical constituents after each porewater volume has passed through the sample. This data will be used to estimate the amount of contaminant adsorption/desorption taking place.

2.3.7.3 As the permeameters are opened after completion of the tests, a visual examination of the samples will be performed. The purpose of the visual examination is to determine whether months of testing has altered the general appearance of the sample. Observations similar to those made in

the pre-test examination (color, texture) will be recorded and the samples will be photographed.

2.3.8 Selection of Backfill Mixture. Results of the compatibility study will be used to select the backfill mixture (constituents and proportions) to be used during construction. Selection will be based on:

- Permeability (lowest)
- Backfill soil alteration (lowest)
- Difference in permeability between tap water and contaminated ground-water (lowest)
- Field constructibility and quality control (greatest)
- Cost (lowest)

The MRD Lab will issue a report on the compatibility study, including all data sheets and calculations. The Final Design Analysis will reference this report and contain a discussion of the results, including the mixture selection.

2.4 Field Vs. Laboratory Permeability. For groundwater modelling purposes, a permeability of 1×10^{-7} cm/sec was assumed for the in situ slurry wall backfill. Using that permeability, a wall thickness of 3 feet, and a head differential of 3 feet across the wall, the calculated time for water to flow through the wall is 20.5 years. However, Darcy's Law takes into account advection of water only, while diffusion and dispersion of contaminants generally causes them migrate faster than water. At very low permeabilities, some studies have shown diffusion and dispersion predominate over advection as a means of contaminant transport. Research has been done to quantify diffusion and dispersion for individual contaminants, but the effects of multiple contaminants is largely unknown. During the life of the wall, water levels inside and outside the cell will be monitored to assure a negative head into the wall. An extraction trench near the north boundary will be used as necessary to maintain a negative head. The designers have decided not to specify a laboratory permeability an order of magnitude lower than the anticipated field permeability. With proper specification and field quality control (i.e., mix design, frequency of QA/QC testing, full mixing) the field permeability should not be severely compromised. If unexpected water flows into the cell, the extraction system will be utilized to remove it.

2.5 Control of Negative Head within Isolation Cell. Removal of ground water trapped within the isolation cell will be required in order to maintain a lower ground-water level within the cell than that outside the cell. This lower level within the cell will assure that no contaminated ground water will migrate out of the isolation cell. Additionally, ground-water recharge into the cell through the cutoff walls and floor of the cell must be considered. The ground-water level drop across the cell is 13 feet, from elevation 5250 at the south to elevation 5237 at the north. Once the cutoff walls are completed, ground water that is trapped within the cell will begin to seek equilibrium. If an equalized horizontal ground-water level were allowed to occur, this level would be at elevation 5244. The equalization process will automatically effect a negative head (a ground-water level within the cell below that outside the cell) from the south cutoff wall northward for a distance of about 250 feet. Since the soil within the cell (and without) has a low hydraulic conductivity,

ground-water flow toward the north cutoff wall will be very slow. Estimated time to reach equilibrium at elevation 5244 without ground-water withdrawal is 16.3 years (the finite difference ground-water model predicts 16 years). The graphical flow net (see discussion section 2.7, Elevation of Ground-Water Flow Diversion) indicates a slight rise in the ground-water level at the south cutoff wall, from elevation 5250 to about 5255, and a slight drop at the north cutoff wall from elevation 5237 to about 5236. Again, as in the equalization process within the cell, the rise and drop of ground-water levels will occur over many years. Because of this, calculations made for the control of the negative head within the isolation cell are based on an initial ground-water elevation of 5250 at the south cutoff wall and elevation 5237 at the north cutoff wall. Initial design ground-water elevations for the negative head are 5243 near the center of the cell and 5234 at the north cutoff wall. Final designed ground-water level is 5234 across the entire cell. Ground-water extraction is normally accomplished by wells. In this case, however, production wells are impractical. Low hydraulic conductivity, impermeable boundary effects, and well interference conditions are factors that make well extraction impractical. Analyses were made for ground-water removal by wells. The results indicate that maximum production from each well would be considerably less than 1.0 gallons per minute (gpm), about 0.08 gpm. Ground-water removal can be accomplished by a single, horizontal drain located 100 feet south of the north cutoff wall. This location of the drain is dictated by the necessity of lowering the ground-water level in the northern one-half of the isolation cell (the southern one-half will automatically adjust to a negative head). The drain is located slightly closer to the north cutoff wall to provide drawdown to elevation 5234 (about 3 feet) whereas the drawdown near the center of the cell must be at elevation 5243 (about 1 foot). Soil conditions at this location favor the emplacement of the drain at elevation 5227. Ground-water flow to the drain was calculated by quantitative methods outlined by Freeze and Cherry (1979). This method involves prediction of ground-water inflow to a vertical excavated face. The model is partially bounded by impermeable boundaries. The equations for this model are time dependent. The drain does not exactly conform to the model, i.e. vertical open face vs enclosed buried drain and impermeable boundaries at the ends of the drain. Given these conditions, estimated maximum ground-water production during the first 230 days of operation is about 19 gpm. This would result in an average drawdown of about 3.5 feet within the isolation cell. Only a minimal volume of ground-water withdrawal is required to maintain a negative head within the cell. The volume of trapped ground water within the cell above elevation 5234 (the designed ground-water level at the north cutoff wall) is 1,111,600 cubic feet. A withdrawal rate of 5 gpm for 230 days and 2.2 gpm for 396 days will lower the ground-water level to the designed elevation of 5234 at the north wall only in about 1.7 years. Since it will require about 11 years to reach elevation 5236.5 outside the cell, lowering the ground-water level in 1.7 years results in a safety factor of 6.4. Ground-water flow into the isolation cell through the cutoff walls and the floor of the cell will be negligible. The rate of flow is calculated to be about 40 cubic feet per day, about 0.2 gpm. The volume of ground water occurring between elevations 5234 and 5235 is 82,225 cubic feet. The time required to raise the ground-water level from elevation 5234 to elevation 5235 due to recharge through the cutoff walls and floor will be about 5 1/2 years.

2.6 Drain Construction. A 6-inch diameter slotted pipe drain shall be installed in a 3-foot wide trench excavated from ground surface to elevation 5228 at the east end of the drain and to elevation 5226 at the west end of the drain. The pipe drain will lead to a lift station located on the west end of the trench. Centralizers will be placed on the drain pipe to assure centering of the pipe within the trench. Fine aggregate for concrete will be used for filter sand. One foot of filter sand shall be placed in the bottom of the trench for bedding for the drain pipe. The filter sand shall be placed around and above the drain pipe to the elevation of the water table, 5239, in 3-foot lifts. Gradation of the filter sand is in accordance with EM 1110-2-1901, Seepage Analysis and Control for Drains. Random backfill shall be placed from the water table to the ground surface. A biodegradable slurry shall be used for the full depth of excavated trench to prevent sloughing of the sidewalls. The biodegradable slurry shall be clean (desanded or new) prior to the placement of the perforated drain pipe. Slurry and biodegraded slurry removed from the trench shall be considered contaminated and shall not be allowed to leave the isolation cell area and will be spread out over the surface.

2.7 Evaluation of Ground-water Flow Diversion. Graphical representations of flow through porous media are called flow nets. Flow nets are an invaluable aid in the solution of various ground-water flow problems. Flow nets are a collection or set of flow lines intersecting a set of equipotential lines. An unlimited number of flow lines and equipotential lines may be drawn, but only a few may be selected to accurately illustrate the general flow condition for the immediate problem. The construction of flow nets involves many intuitive deductions and may be considered an art rather than a science. However, if fixed conditions are the rule at all points of a boundary of a saturated soil mass, a flow net is uniquely determined. That is to say, one and only one solution exists. If, however, there is a change in boundary conditions, a new unique solution will then exist, but it may take a long interval of time to achieve steady state conditions. One flow new and one ground-water flow diagram were constructed to determine the possibility of diversion of contaminated ground water into non-contaminated or less contaminated areas. The only information a priori for the flow net were ground-water levels measured in 1989. A pre-construction flow diagram was constructed for comparison with a post-construction flow net. Although a uniform saturated thickness was assumed for the construction of the flow net, the error introduced will only have minimal effect on the study. The resultant upgradient and downgradient potentials from the post-construction flow net were estimated based on the configuration of a priori equipotential contours. The study indicates that there will be a rise in ground-water level at the south wall of the slurry trench (upgradient) of about five to six feet (elev. 5255 to 5256). Conversely there will be a drop in ground-water level at the north wall of the slurry trench (downgradient) of about one foot (elev. 5236). The finite difference ground-water flow model predicts a rise only to elevation 5252.5 south of the isolation cell, and a drop to elevation 5237 north of the cell. Flow lines of the net indicate that ground-water flow will be diverted about equally east and west of the containment cell. The ground-water flow will parallel the sides of the containment cell and then will converge north of the containment cell, again nearly following the flow path that the ground-water regime had prior to the construction of the containment cell. It is concluded that there will be no significant diversion of the ground-water flow regime.

2.7.1 Ground-water Flow Model. A finite difference ground water flow model was used to simulate ground water flow in the alluvial aquifer of the lime settling basin area at Rocky Mountain Arsenal. Software used to develop the model is part of the Well Field Simulation Package developed by Hall Groundwater Consultants. The model is based on a finite difference computer model developed by Prickett and Lonnquist (1971) which has been modified by Hall Groundwater to run on IBM PC compatible computers.

2.7.1.1 The data base used to develop the model was drawn from a bedrock elevation map and a groundwater elevation map prepared by Woodward and Clyde Consultants. Data required for the finite difference grid nodes were extrapolated from these two maps. Permeability of $2.12 \text{ gallons/day/foot}^2$ ($1 \times 10^{-4} \text{ cm/sec}$) was used for the model. This value was confirmed by a series of slug tests recently conducted by Woodward and Clyde Consultants. The storativity used for the model was 0.2. Modeling was conducted in three phases. Two finite difference grids were used. The first two phases of modeling were based on a symmetrical, 29x29 grid. Grid spacing varied between 200 feet, at the outer margins, to 50 feet within the area of the lime settling basin. The third phase of modeling used a 29x26 grid with a constant node spacing of 25 feet.

2.7.1.2 Initial modeling used only data input from the groundwater elevation and bedrock elevation maps in order to calibrate the model. Boundary conditions were varied during this phase of modeling to most closely approach the current groundwater conditions at the Lime Settling Basin. The closest approximation to actual conditions at the Lime Settling Basin Site was achieved by setting up all of the boundaries as constant head boundaries. In the Hall Groundwater Model a constant head boundary is modeled by using an extremely high storativity value in the nodes which define the boundary. A value of 2×10^{12} was used for this modeling.

2.7.1.3 The model was calibrated by simulating existing head conditions in the unconfined aquifer. After the model was calibrated, the slurry wall was input into the model by reducing the permeability at the nodes which define the position of the wall by three orders of magnitude relative to background. A value of $.00212 \text{ gallons/day/foot}^2$ ($1 \times 10^{-7} \text{ cm/sec}$) was used for the slurry wall permeability. Boundary conditions were the same as in the first phase of model runs. These model runs were used to determine the effects of the slurry wall on the local groundwater flow system and to compare the computer generated flow system with a flow net which was generated, based on hand calculations, prior to the start of computer modeling. The match between the computer generated flow system and the flow net matched over most of the model. There were slight differences between the two flow systems near the northern boundary of the slurry wall because of a minor difference in extrapolated groundwater head contours used in developing the two models. The computer simulation indicates a slight rise in ground-water level upgradient of the isolation cell (about 2.5) feet, and a slight drop downgradient (about 2 feet). The ground-water is shown as equalizing within the isolation cell at about elevation 5245. Total time to attain stabilized ground-water flow after construction of the isolation cell is about 11 1/4 years.

2.7.1.4 The third phase of modeling was concerned only with flow within the confines of the slurry wall. Model boundaries were determined by the position of the slurry wall over much of the model, where the boundary was set as a no flow boundary. In the northern part of the model boundaries were set up as constant head boundaries. While a constant head boundary in this location is not realistic, the relative impermeability of the slurry wall makes the effects of a constant head boundary negligible, considering it was only with the interior of the slurry wall area that this phase of modeling was concerned. Results of the modeling indicate that the groundwater will stabilize within the isolation cell at about elevation 5243 in about 19 1/2 years. This model simulation does not consider any ground-water removal from the isolation cell.

2.7.1.5 A row of pumping wells was input within the slurry wall area 100 feet south of the north slurry wall to try to simulate the effects of a drain. Ten wells were simulated and at all pumping rates ground-water withdrawal was so rapid the wells were considered to be pumped dry by the model. The model run of a pumping rate of 0.10 gpm resulted in a drawdown to about elevation 5235.5 at the north wall of the isolation cell. The pumping dry of the well is a result of a combination of boundary effects and the low permeability of the material that makes up the aquifer.

2.8 Evaluation of Bedrock Aquifer Contamination. A piezometer cluster is located 50 feet east of the east wall of the isolation cell at coordinates N 2,185,002; E 181,126. This cluster has separate piezometers installed in the alluvium and the Denver "A" sandstone unit which is 34.5 feet below the top of bedrock. Ground-water elevation in alluvium was measured at elevation 5248 and ground-water elevation in the Denver "A" sandstone unit was measured at elevation 5254. These measurements were made in April 1990. The measurements indicate there is a downward hydraulic gradient into the Denver formation of 0.032 ft/ft. The designed elevation of ground-water within the isolation cell to maintain negative head is 5234. This will result in an upward gradient from the Denver "A" sandstone unit into the isolation cell of 0.26 ft/ft. Since the gradient is upward into the cell, there will be no contamination of the Denver Formation from the isolation cell. Ground-water flow from the Denver Formation into the isolation cell is calculated to be about 25 cubic feet per day.

2.9 Alignment of Slurry Wall. It was observed during site visits and reviews of current aerial photography and 1940's topographic maps, that the extent of sludges deposited outside the Lime Settling Basins could be possibly up to 10 feet in depth. Information from the investigative borings confirm that sludges deposited north of the Lime Settling Basins are approximately 7 to 9.5 feet in depth, approximately 1 to 2 feet in depth on the area west of the basins, and 1 to 2 feet in depth on the south side area of the basins. The option to place the slurry wall around the existing Lime Settling Basins only, did not provide enough storage capacity to contain all the excavated contaminated sludges outside the Lime Settling Basins. It was felt that the deposited sludges outside the existing Lime Settling Basins should also be contained within the slurry wall isolation cell, as they could be considered a contaminant source. The alignment of the Slurry wall around the Lime Basins was extended to the north in order to contain more contaminated in-situ sludge material, and provide for more storage capacity of excavated contaminated soils. The area contained by the slurry wall isolation cell is directly adjacent to Basin A, and therefore

the slurry wall will be constructed through contaminated soils. The slurry wall will not completely surround the contaminated area, but it will contain the contaminated source area of the Lime Settling Basins.

2.10 Slurry Trench Width and Depth. The width of the slurry trench will be 3 feet. The depth of the slurry trench was estimated to have a maximum depth of 35 feet from the ground surface and an average depth of 28 feet. The trench will be keyed into the Denver Formation claystone 2 feet. Establishment and maintenance of a negative head within the isolation cell will only require that the bottom of the trench be excavated through the overburden material. Emplacement of the slurry wall through the overburden will eliminate excessive recharge into the isolation cell. Only slight leakage, if any will occur through the claystone and into the cell when the slurry wall is keyed two feet below the top of bedrock. Whenever uncemented, loose fine-grained sandstone is encountered at the top of bedrock, it will be excavated to the depth of cemented, hard fine-grained sandstone or claystone whichever is encountered first. The cemented sandstone has a low permeability, 1.0×10^{-5} cm/sec or less, and will not appreciably affect the recharge into the isolation cell. Average depth of excavation into the Denver Formation is anticipated to be just slightly greater than two feet.

2.11 Construction.

2.11.1 Work Zones. The exclusion, contaminant reduction, support, and staging zones are shown in the drawings. The support zone is located west of the Lime Basins, north of the RMA Fire Station. The staging and contaminant reduction areas are located just east of the support zone. Arsenal personnel do not want heavy dump trucks loaded with off-site borrow soil to access the site via December 7 Avenue because the trucks might damage the pavement. Therefore a gravel access road will be built accessing the site from the east. To keep that road clean, empty trucks will exit the site via the southwest.

2.11.2 Grading. Minor grading will be necessary to provide a construction platform for the slurry wall installation. The work platform will be 40 feet wide and have a maximum slope of 1% along the slurry wall centerline. Since the existing surface soils are contaminated, the work platform will be covered with 12" of clean borrow material in order to provide a clean area on which to work.

2.11.3 Excavation. Excavation of the trench will be made with a large track mounted extended-reach backhoe or by a crane-mounted slurry-trench clamshell. The trench is kept from collapsing by the bentonite slurry. Water for slurry mixing operations is available from the water truck filling facility at the RMA Fire Station. Excavated materials will be placed in the isolation cell, if it is determined during compatibility testing that the material is unsuitable for backfill. The Contractor will have the option of performing the overexcavation of contaminated materials on the south, north, and west of the Lime Basins either before or after construction of the slurry trench.

2.11.4 Sequential Construction Evaluation. An evaluation of sequential construction of the slurry trench has been made to determine if a significant lowering of the ground-water table will occur during the construction

of the isolation cell. Once the south cutoff wall has been constructed, ground-water lowering will occur on the north side (eventually in the trapped portion of ground water within the cell). The greatest lowering (or escape out of the cell) of ground water as a result of sequential will occur if excavation is started at the north end of the east cutoff wall. The excavation must then proceed southward for excavation of the east wall thence continuing around the isolation cell until completion of the cell is made by connecting to the north end of the east wall. It is calculated that 5,310 cubic feet of ground water will escape the isolation cell because of this sequential construction. This amount is insignificant when compared to the amount that must be removed (about 1,111,600 cubic feet) for maintenance of a negative head. Since sequential construction will place a restriction on the contractor's operations, which is not cost justified, sequential construction of the slurry wall will not be specified.

2.11.5 Slurry Preparation. The Contractor will choose the method of mixing slurry. It is anticipated slurry will be mixed by a bulldozer on a concrete pad or by a high velocity mixer. The method, design, and rationale for the slurry mixing operation will be a Category I submittal.

2.11.6 Stability. The stability of a 28-feet deep (average) slurry trench is not anticipated to be a major concern, since trenches over 100 feet deep have been successfully completed by others.

2.11.7 Backfilling. Backfill material will be blended and trucked to the trench where it will be moved into the trench with a bulldozer. Blending operations are typically done with a pug mill operation or by mixing with a bulldozer on a concrete mixing pad. The Contractor will select the method of blending the backfill material. Blending operations will be done in the Contractor's staging area. The method, design, and rationale for the chosen mixing method will be a Category I submittal.

2.11.7.1 Backfill Rate. The Corps guide spec states the toe of the slope of the trench excavation shall not precede the toe of the backfill slope by less than 50 feet or more than 105 feet (although those values may be changed). Xanthakos states there is no real reason for specifying somewhat arbitrary distances, and says that the minimum distance would be the distance the Contractor would need to properly clean the bottom of the trench, which he states is approximately 30 feet. EPA-540/2-84-001 recommends the distance be minimized for stability reasons, but states it may be up to 200 feet. D'Appolonia recommends having slurry in the trench for at least 24 hours prior to placing backfill to allow for proper filter cake formation. None of the references checked listed any method or reason for specifying a maximum distance between the toe of the excavation slope and the toe of the backfill slope. Therefore the specification states that the distance will be kept to the minimum value which will allow both cleaning of the trench bottom and a minimum 24 hours between slurry placement and soil-bentonite backfill placement. Because a formal stability study was not undertaken, it will also be specified that the distance shall not be greater than 105 feet without the approval of the Contracting Officer.

2.11.8 **Bends in Alignment.** The slurry trench will be overexcavated at corners to assure the full depth of the trench for at least 2 feet outside the isolation cell.

2.11.9 **Compacted Clay Trench Cover.** To prevent the soil-bentonite backfill mixture from desiccating, the top one foot (cut out of the work platform) will be covered with compacted clay obtained from the previously mentioned clay borrow area used during Basin F remediation. The cover will be 8 feet wide and will be placed between 2 and 4 days after backfilling. At this time it may only be compacted with a backhoe bucket or small hand-operated smooth drum roller because the soil-bentonite backfill may still be somewhat soft. Two weeks after backfilling, the cover will be recompacted with standard compaction equipment and any areas of settlement will be filled in with more clay material and compacted. At this time the Contractor will excavate two areas of the trench to be used as heavy equipment crossings during subsequent construction. The crossings consist of 2 18-inch and one 12-inch layer of compacted clay separated by geotextiles as shown on the plans. During construction of the vegetative cover, the compacted clay wall cover and the work platform will be covered by random fill, topsoiled, and vegetated.

2.11.10 **Quality Assurance/Quality Control.** QA/QC testing of materials is given in Tables 1 and 2 of specification section 02214, Soil-Bentonite Slurry Trench Cutoff. In addition, soundings to determine the top of bedrock, trench bottom, and backfill slope will be made at horizontal intervals of 20 feet. Undisturbed samples of the completed trench for permeability testing will be taken every 400' lineal feet.

2.11.11 **Abandonment of Existing Wells and Piezometers.** Wells and piezometers 36055A, 36055B, 36058, 36059, 36076, 36167, and 36194 in the Lime Settling Basins area and 01503 and 01504 in the M-1 Basins area will be abandoned. The abandonment is required because the wells and piezometers are located in the construction area. The abandonments will be accomplished prior to other construction activities. Abandonment will be in accordance with RMA policy. Concrete pads will be broken and removed; surface protective, steel casings will be pulled and removed; and the remaining PVC casings and screens will be overdrilled with a hollow stem auger. A cement-bentonite grout mixture of 94 pounds of Type II Portland cement, 3 pounds of powdered bentonite and a maximum 8 gallons of water. The grout will be placed in the overdrilled hole by tremie pipe beginning at the bottom and continuing to the ground surface while the auger sections are removed. A complete record of original well installation data and well abandonment procedures and data will be made for each abandoned well and piezometer.

2.12 LONG TERM MONITORING. Long term monitoring for ground-water quality and piezometric levels will be required to assure the isolation cell is performing as designed. This will be made possible by installation of monitoring wells and piezometers. Monitoring wells will be placed upgradient, and downgradient of the isolation cell and crossgradient on each side of the isolation cell. These wells will be placed near the center of the alignment of the east, south, west, and north walls of the isolation cell. One monitor well will also be placed inside the isolation cell near the center of the north wall. Piezometers will also be placed near the centers of the alignments of the walls

and will be located very close to the walls. The piezometers will be inside and outside (mirror imaged) the isolation cell to closely monitor for the maintenance of the negative head within the cell. The locations of monitor wells and piezometers are shown in the Contract drawings.

2.12.1 Construction and Installation of Monitor Wells and Piezometers. Construction and installation of monitoring wells and piezometers will follow procedures outlined in MRD Policy Letter #90-001. The construction will consist of installation of 4-inch ID (for monitor wells) and 2-inch ID (for piezometers) PVC, threaded casings and continuous wire wound type screens; end caps; no grease or oils (other than vegetable oils) will be allowed. Designed sand filter packs, bentonite seals and cement-bentonite grout will be required. Well and piezometer development will be required and turbidity of the water will be measured after development has been completed. Diagrams of the monitor wells and piezometers are shown in the Contract drawings.

3. VEGETATIVE COVER (LIME SETTLING BASINS).

3.1 Design. The cover to be constructed is intended to be a vegetative cover over the Lime Settling Basins. This cover will minimize infiltration and promote drainage away from the Lime Settling Basins. The substantive standards contained in 40 CFR 264.310, specifically those requirements contained in subsections a(2)-(4), and b(1) and (4), describe the necessary standards relevant to this cover. The cover will consist of 12" of compacted fill material and topped with 6" of topsoil. The cover will have a minimum slope of 2 percent to promote drainage. The cover will be seeded with an appropriate seed mixture to minimize erosion to less than 2 tons/acre/year. The Hydrologic Evaluation of Landfill Performance (HELP) Model has been used to determine rainfall infiltration rates through the vegetative cover. Infiltration rates are currently estimated to be less than 0.012 inches/year.

3.2 Pond Dewatering and Filling. The Lime Settling Basins will have approximately 2 acre-feet of water removed prior to fill material placement. Pending the results of the water quality testing, the water will be drained into the drainage located to the northeast of the Lime Settling Basins, which eventually flows into Basin A. Impacts of the additional water to Basin A will be evaluated. Other options include the construction of a lined evaporative pond which could be used to store the water until evaporated. The evaporative pond would be lined with a geomembrane to prevent infiltration into the ground. Once drained, the Lime Settling Basins will be filled with clean fill material up to the existing ground water elevation of approximately 5248.

3.3 Contaminated Excavations. Contaminated soils outside the slurry wall containment cell, located to the south, west and north, will be excavated and placed inside the containment cell. Newly placed contaminated soils will be placed above ground-water level. Dust control will be critical during all excavations.

4. SHEET PILE CUTOFF WALL (M-1 SETTLING BASINS)

4.1 Criteria. The Decision Document requires the containment system consist of a 360 degree subsurface barrier around the M-1 Settling Basins,

vitrification of soil/sludge by introducing an electric current through an array of electrodes, an offgas treatment system for capture of organics, air monitoring during implementation, and ground-water monitoring to evaluate the continued effectiveness of this ISV alternative. Steel sheet piling was determined to be the preferred barrier to be installed. Sheet piling will allow quick, easy installation, and provide temporary containment of the ground-water during the ISV process. The sheet piles will be removed after vitrification.

4.2 Location and Alignment. Information supplied by Geosafe Corporation, the ISV vendor (Application and Evaluation Considerations for In Situ Vitrification Technology: A Treatment Process for Destruction and/or Permanent Immobilization of Hazardous Materials, April 1989), a very steep thermal gradient, approximately 150-200 degrees C per inch, precedes the advancing melt surfaces. Typically, the 100 degrees C isotherm is less than 1 foot away from the molten mass. It was decided to locate the sheet pile 10 feet away from the design limits of vitrification in order to provide adequate room for ISV operations.

4.3 Key Depth. The sheet piles will be driven one foot into the bedrock surface, or until refusal. The boring logs along the sheetpile alignment show bedrock at a depth of 9 to 14.5 feet, generally 9 feet on the south boundary increasing to 14.5 feet on the north boundary.

4.4 Compatibility With Contaminated Groundwater. The sheet piling is a temporary measure to reduce the flow of groundwater into the area prior to and during vitrification primarily to save electricity (and therefore money) by reducing the amount of water that is evaporated during vitrification. The Rocky Mountain Arsenal Project Manager (PMRMA) has indicated the time between sheet pile placement and vitrification will be on the order of a few months. For this reason, compatibility of the steel with the contaminated groundwater is not considered to be a problem and no compatibility testing will be done.

4.5 Pile Sizing. The pressures against the pile and bending moment of the pile are not anticipated to be major concerns, since no excavation will take place inside the cell. The vitrification process does result in a volume decrease and therefore subsidence of the ground surface, but experience at other vitrification projects has shown this to be only a few feet. Since the vitrified mass will be several feet from the sheet piling, the full amount of subsidence will not take place against the sheetpiling but several feet away from it. After vitrification the subsided area will be filled in with clean soil to avoid leaving a depression in the area. Therefore the major consideration in pile sizing is to survive the driving process. Piles used for the cutoff wall will be PZ22 steel sheetpile.

4.6 Construction.

4.6.1 Sheet Pile Installation.

4.6.1.1 Work Zones. The work zones for construction are shown on the drawings. The Exclusion Zone extends 10 feet outside the sheet pile wall centerline. The Contractor will store his equipment and perform all operations from the inside the area to be surrounded by the sheet pile. The Arsenal does

not want workers straying outside the exclusion zone since that area is also contaminated. A note has been placed on the drawings stating that workers are not to go outside of the work zones.

4.6.1.2 Vibrations. There is concern vibrations produced during pile driving may damage adjacent structures. Of major concern to the Arsenal is an underground rigid asbestos water line located just south of December 7 Avenue (about 50 feet north of the northern pile boundary). Research and experience in the field of soil dynamics has shown that peak particle velocity is the parameter most closely related to vibratory damage of structures. As long as peak particle velocity is less than 1 or 2 inches per second damage will not occur. One inch per second is about the lowest vibration most people can perceive. Vibratory hammers produce much less vibration than impact hammers, and this job will be specified as vibratory hammer only. Figure 7 of "Construction Vibrations: State-of-the-Art" (ASCE Journal of the Geotechnical Engineering Division, February 1981) shows that at a distance of 50 feet, the peak particle velocity produced by a vibratory hammer will be approximately 0.3 inches per second. The project plan calls for demolition of above ground structures inside the sheet piling and up to 10 feet outside the sheet piling. According to Figure 7, at a distance of 10 feet a vibratory hammer will produce a peak particle velocity of about 2 inches per second. It is possible structures between 10 and 25 feet away from the driving might receive some damage. However, these structures are not in use now and most probably never will be used again. Any damage that might occur would be minor concrete cracking, as the peak particle velocities are not high enough to cause adjacent structures to collapse. Therefore, it is not anticipated vibrations will be a problem beyond 10 feet. As a precaution, four settlement monuments will be installed in the area prior to driving. Two will be located near the previously mentioned water line about 50 feet north of the north side of the sheetpile, one will be located near the overhead pipe just north of tank T 66, and one will be located just north of the concrete structure 561. The latter two monuments will be approximately 15 feet from the east and south boundaries of the sheetpile respectively. These monuments shall be monitored daily for the first several days of driving and when driving is occurring close to the monuments. If settlement is observed, the Contractor may have to adjust his operations. After the sheet piles are driven and in place, the piles will be cut off to be flush with the existing ground.

4.6.2 Sheet Pile Removal. The sheet piles will not be removed under this contract, but will be removed by others in the future.

4.6.3 Abandonment of Existing Wells and Piezometers. Abandonment of existing wells and piezometers in the construction area will be in accordance with RMA Standard Operating Procedures and/or MRD Policy Letter #90-001.

4.7 LONG TERM MONITORING. Long term monitoring of the M-1 Basins will be done by others and is not required in this contract.

5. CIVIL: GRADING, PAVING, AND DRAINAGE. (LIME SETTLING BASINS)

5.1 DESIGN REFERENCES. The following references were used in preparing the grading, paving, and drainage design:

5.1.1 Department of the Army and Air Force Technical Manuals.

TM 5-820-1 88-5, Chap 1	Surface Drainage Facilities for Airfields and Heliports (Aug 87)
TM 5-820-4 88-5, Chap 4	Drainage for Areas Other Than Airfields (Oct 83)
TM 5-822-2 88-7, Chap 5	General Provisions and Geometric Design for Roads, Streets, Walks, and Open Storage Areas (July 87)
TM 5-822-5 88-7, Chap 3	Flexible Pavements for Roads, Streets, Walks, and Open Storage Areas (Oct 80)

5.1.2 Department of the Army Technical Manuals (TM).

5-820-3	Drainage and Erosion Control, Structures for Airfields and Heliports (Jan 78)
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5.1.3 Engineer Manuals (EM).

1110-2-2902	Conduits, Culverts and Pipes (Mar 69)
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5.1.4 NEENAH Foundry Company Publication.

Inlet Grate Capacities for Ponded Water
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5.1.5 Engineering Technical Letter (ETL)

1110-1-140	Pavement Design for Roads, Streets, and Open Storage Areas (July 88)
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5.2 GRADING. The following criteria was used to develop site grading.

5.2.1 Crown grade of 2 percent.

5.2.2 Maximum desirable ramp grade of 7 percent. Absolute maximum
ramp grade of 10 percent for short distances only.

5.2.3 Minimum grade of 1 percent for overlot grading for
cohesionless sandy soils and 2 percent for cohesive soils or
turfed areas.

5.2.4 Minimum ditch grade of 0.3 percent.

5.2.5 Maximum foreslopes of 1V on 4H and backslopes of 1V on 3H.

5.3 FLEXIBLE PAVEMENT. The temporary construction access road was designed for a drivable surface for construction equipment and for dust control.

5.3.1 Traffic consists of the following vehicles:

1) various construction equipment including dump trucks and earth moving equipment.

5.3.2 Strength Method. (Non-Frost Design)

Class = E
Category = IV
Design Index = 4
CBR = 7
Design Thickness = 12 inches

5.3.3 Recommended Pavement Section.

6-inches Crushed Rock Surface Course
6-inches Crushed Aggregate Base Course
6-inches Compacted Subgrade (95% maximum density)

5.4 DRAINAGE. Drainage was designed in accordance with AFM 88-5, chapter 1, TM 5-820-3, and TM 5-820-4. The existing storm drainage system was extended and routed around the lime settling basin. The 30-inch diameter Reinforced Concrete Pipe (RCP) was partially removed, to clear the slurry trench, capped, and abandoned in place. This 30-inch RCP was previously abandoned and capped upstream and carried no storm discharge. The 24-inch RCP was removed to the last downstream manhole and extended from this location. Due to the small drainage area added to this drain line no increase in pipe size was required.

5.4.1 Storm Drain Pipe. Storm drains were designed to withstand earth dead loads as well as H-20 or HS-20 highway live loads.

5.4.1.1 Pipe Materials. Reinforced Concrete Pipe (RCP) was chosen to match the existing storm drainage system.

5.4.1.2 Pipe Joints. Watertight pipe joints were required to prevent infiltration of soils through the joints due to the presence of ground water at or above the pipelines and the use of erodible backfill materials.

5.4.2 Inlet Capacity.

5.4.2.1 Area Inlets. The capacity of area inlets in a sump condition was determined using the nomograph in the NEENAH Foundry Company publication entitled "Inlet Grate Capacities for Ponded Water" for a NEENAH type R-6118 catch basin frame and grate.

6. WATER SUPPLY AND WASTEWATER COLLECTION. (LIME SETTLING BASINS)

6.1 DESIGN REFERENCES. The following references were used in preparing the water supply and wastewater disposal design:

6.1.1 Department of the Army Technical Manuals (TM).

TM 5-814-1	Sanitary and Industrial Wastewater Collection-Gravity Sewers and Appurtenances (Mar 85)
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TM 5-814-2	Sanitary and Industrial Wastewater Collection-Pumping Stations and Force Mains (Mar 85)
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6.1.2 National Standard Plumbing Code (1983).

6.1.3 Recommended Standards for Sewage Works by the Great Lakes-Upper Mississippi River Board of State Sanitary Engineers (1978 Edition).

6.2 GENERAL. The work under this project consists of containing and pumping contaminated water from within the perimeter of the slurry wall at the Lime Settling Basins. It was determined that the ground-water should be artificially depressed within the confines of the slurry wall in order to prevent the migration of contaminated ground-water away from the project boundary.

6.2.1 The Containment and Pumping system consists of a 36-inch diameter lift station, 1 HP sump pump and 660 feet of 2-inch forcemain. The system is designed to contain the estimated flow rate of 5 gpm from the perforated groundwater collection drain and pump it to the CERCLA Water Treatment Plant. The volume to be pumped is considered a finite amount, necessary to provide a negative head/gradient within the settling basins.

6.2.2. Constructibility was the most significant consideration in the design. The collection drain is to be placed approximately 25 deep in heavily contaminated, water saturated, sandy clays which are not stable enough for normal trench excavation. Therefore, the piping will installed by a slurry method. A standard concrete type lift station would be very difficult to construct in the material at the depths required. Therefore, a 36-inch diameter polyethylene pipe will be used for the pump chamber because it can be attached to the collection piping above ground and easily lowered into the trench through the slurry.

6.2.3. The 36-inch diameter pump chamber is 33 feet deep and is designed to provide 5 feet of storage volume between the invert of the collection drain and the bottom of the pump chamber. The 5 feet of depth amounts to 265 gallons of storage. At a inflow rate of 5 gpm, the storage volume will take approximately 50 minutes to fill allowing the pump adequate time to cool. The design requires a 1 HP sump pump to operate at 22 gpm at 52 feet of head. With the storage volume available the pump will operate for approximately 12 minutes

during each pump cycle. The pump is controlled by three float switches suspended in the lift station. The lowest float switch is the pump "off" control, the second float switch located at the elevation of the collection pipe invert is the pump "on" control and the highest float switch located 1 foot above the invert of the collection pipe is the "alarm" switch. These switches will automatically control the pump operation. However, a manual lift station switch will be provided at the CERCLA Water Treatment Plant to shutdown or turn on the lift station controls. In addition, any alarms at the lift station will be monitored at the CERCLA Water Treatment Plant. The manual control and alarm monitoring will be part of the CERCLA Water Treatment Plant project, however, provisions have been made in this design to accommodate this work.

6.2.4. Approximately 1000 feet of 2-inch forcemain is needed to convey the contaminated water from the lift station to the CERCLA Water Treatment Plant. However, only 660 feet of 2-inch forcemain will be provided under this project because not enough survey is available for entire length of the pipe and the exact location for the CERCLA Plant has not been finalized. The remainder of the piping to the CERCLA Plant will be provided as part of the CERCLA Plant project. The route of the forcemain will be easily identified by new overhead power lines, required to power the lift station, running immediately parallel to the forcemain and the end of the forcemain will be identified with a marking post.

6.2.5. The lift station is designed to facilitate all maintenance without entering the pump chamber. Discharge piping is connected directly to the pump and runs directly up to a union at the top of the manhole which can be disconnected to raise pump and piping from the pump chamber during maintenance activities.

6.3 M-1 BASIN DESIGN. Also incorporated within this design package is the relocation of a fire hydrant and capping of various utilities near the M-1 Basins on the south side of December 7th Avenue to facilitate operability of the ISV process (to be designed/constructed in FY 90 thru 93).

7. CHEMISTRY. No chemical analysis is required by the Contractor other than outlined in the Site Safety and Health Plan (SSHP).

8. ELECTRICAL (LIME SETTLING BASINS)

8.1 General. The electrical design is based on the following codes, standards, publications, etc:

- 8.1.1 National Electrical Code (NEC) NFPA No. 70-1990
- 8.1.2 Life Safety Code NFPA No. 101-1990
- 8.1.3 National Electrical Safety Code (NESC) ANSI C2-1990
- 8.1.4 Architect Engineer Instruction Manual AEIM 14(Rev. July 1990)

8.2 Scope. This design will provide electrical power for the ground water waste pump located at the lime settling basins.

8.2.1 The new basin M-1 cutoff sheet pile walls will be located near existing 13800V 3-phase lines on both the north and west side of the cutoff wall. These lines will be either removed or relocated by the Rocky Mountain Arsenal's electrical distribution contractor.

8.2.2 A new aerial extension of the existing 13800V 1-phase line will be installed from the existing line west of "D" Street and routed to the west edge of the lime settling basins" cap.

8.2.3 A single phase pole mounted transformer would be provided at the end of the new aerial extension. The transformer will have fused cutoff switch and surge arresters. The transformer secondary will be 240/120V and will be routed above the cap in plastic conduit with an equipment ground. The conductor will be sized for load and distance from transformer to load (voltage drop considerations). A combination circuit breaker type motor starter with weather proof enclosure will be locate near the pump.

8.2.4 A ground fault circuit interrupter 120V, 20 ampere receptacle will be located on or near the combination motor starter.

8.3 Specifications. The following guide specifications will be edited for this project: (See Attachments)

Note that section CEGS-16415-OD would be retitled Electrical Work. In addition, section CEGS-16401-OD will be provided.

9. Health and Safety. The specifications for the remedial action will present requirements to ensure that the Contractor performs the work in compliance with applicable regulations, especially 29 CFR 1910.120, "Hazardous Waste Operations and Emergency Response". The specifications will require the Contractor to maintain a Safety and Health Program and to prepare a Site Safety and Health Plan (SSHP) covering all work to be performed under the construction contract. The paragraphs below describe background information and decision logic involved in determining specific requirements that will be included in the specifications.

9.1 Site description and contamination characterization.

9.1.1 Site description. (General) RMA occupies more than 17,000 acres (approximately 27 square miles) in Adams County, directly northeast of metropolitan Denver, Colorado. The property was purchased by the US government in 1942 for use in World War II to manufacture and assemble chemical warfare materials, such as mustard and lewisite, and incendiary munitions. Starting in the 1950s, RMA produced the nerve agent GB (isopropyl methylphosphonofluoridate) until late 1969. A significant amount of chemical warfare materials destruction took place during the 1950s and 1960. From 1970 to the early 1980's, RMA has primarily been involved with the destruction of chemical warfare materials. In addition to these military activities, major portions of the plant facilities were leased to private industries, including Shell Oil Company, between 1947 and 1982, for the manufacture of various insecticides and herbicides.

9.1.1.1 M-1 Settling Basins Description: The M-1 Settling Basins are located in the South Plants area, just south of December 7th Avenue along the northern edge of the northwest quarter of Section 1. The basins and the berms surrounding them, all of which are now buried and partially built upon, occupy an area of approximately 34,500 square feet.

9.1.1.1.1 The M-1 Settling Basins were constructed to treat waste fluids from the lewisite facility. Two basins were constructed in 1942, and a third basin was constructed in 1943 when the original two filled with solids. All three were unlined, and each measured approximately 90 feet wide, 115 feet long, and 7 feet deep. In addition to the waste fluids from the lewisite disposal facility, the basins may have contained lesser amounts of waste materials from alleged spills within the acetylene generation building, the thionylchloride plant, and the arsenic trichloride plant, which may have been routed through floor drains and the connecting piping to the basins. The basins also received a considerable amount of mercuric chloride catalyst, possibly from a spill.

9.1.1.1.2 The liquids discharged into the basins first passed through a set of reactor towers where calcium carbonate was added, then through a wood trough into the M-1 settling basins where the arsenic precipitated out of solution. The elutrate was decanted through an 18-inch diameter pipe to the Lime Settling basins in Section 36 where final treatment occurred, prior to being routed to Basin A.

9.1.1.1.3 The M-1 Settling basins were backfilled, probably in 1947, and are now covered with soil. Portions of the basins are covered with structures. The facilities that surround the M-1 Settling Basins area were used in the manufacture of bicycloheptadiene until 1974.

9.1.1.2 Lime Settling Basins Description: The Lime Settling Basins, located in Site 36-4, are in the southwestern portion of Section 36 at RMA and consist of three unlined basins, each approximately 1 acre. The boundaries of the Lime Settling Basins include berms that surround the basins as well as associated materials that separated the basins. The total area of investigation is approximately 210,000 square feet and has an average surface elevation of 5,255 feet above mean sea level (MSL).

9.1.1.2.1 The Lime Settling basins were constructed in the early 1940's to remove arsenic from South Plants wastewater by precipitation. Wastewater was treated with lime at the site to precipitate metals and reduce arsenic concentrations generated by the manufacture, and later the demilitarization, of lewisite. The basins were also constructed to receive wastewater from the industrial activities at the South Plants until the chemical sewer was constructed in the early 1950s. All wastewater originating from the South Plants area was channeled through the Lime Settling Basins prior to entering Basin A. This water flowed through an underground sewer and into a ditch along the south side of the basins. From the ditch, flow into the basins was controlled. Materials possibly contained within the basins include a reported spill of 500 gallons of mercury catalyst and the disposal of approximately 150 drums of mustard in the basins between 1959 and 1960. Reports also note that

the mustard may have been neutralized, and that the term "drum" refers to a volume and not that the material was disposed of in drums.

9.1.2 Contamination characterization. Previous field sampling has shown contamination to be present in soil, ground-water and surface water at the Lime and M-1 Settling Basins. Classes of chemicals detected include organochlorine pesticides, organosulfur compounds, volatile organic compounds, metals, and agent degradation products. Additional field work is currently underway to further characterize the contamination at these sites. Soil and water samples are being collected and analyzed for volatiles, semi-volatiles with DBCP, organochlorine pesticides, thioglycol, ICP metals, arsenic and mercury. A detailed list of chemical names, concentration ranges and media in which found will be included after the latest analytical results are received.

9.2 Hazard Assessment and Risk Analysis. The remedial action for the Lime and M-1 Settling Basins will involve a number of tasks/operations. The following is a preliminary list of general tasks/operations. A more detailed description will be available later in the design process.

- Mobilization and Site preparation
- Demolition of structures
- Abandonment, installation of monitoring wells
- Construction of slurry walls
- Installation of drain/trench system
- Excavation of sludge from outside wall
- Construction of clay "cap" (cover).
- Construction of sheet pile cutoff wall
- Seeding
- Demobilization and site closeout

The following is a list of general hazards that may be encountered. As the tasks are further defined, detailed hazard analyses will be conducted for each task.

Physical Hazards

Normal outdoor work hazards: slips, trips, falls, etc.

Normal construction hazards:

- Moving equipment
- Use of power tools
- trenching hazards
- falling objects

Noise

Heat/cold stress (depending on the time of year)

Biological Hazards

Poisonous and/or thorny vegetation

Insect bites, stings

Snakes

Diseases carried and transmitted by rodents

Chemical Hazards

volatile halogenated solvents
volatile aromatic solvents
mustard agent-related organic compounds
herbicide-related organosulfur compounds
GB agent-related organic compounds
organochlorine pesticides and pesticide-related compounds
metals

9.3 Accident Prevention. The contractor will be required to follow accident prevention procedures outlined in the USACE Safety Requirements Manual (EM 385-1-1). Some of these requirements (i.e. training, hazard analysis, ...) are addressed in other sections of this Design Analysis. The SSHP prepared by the Construction Contractor will serve as the Accident Prevention Plan (APP) and Activity Hazard Analyses (phase plans) described in EM 385-1-1, thus a separate APP will not be required. Accident reporting requirements will also be addressed.

9.4 Staff organization, qualification, and responsibilities. The contractor will be required to develop an organizational structure that sets forth lines of authority, responsibility, and communication. Part of this organization will be personnel responsible for oversight and implementation of the health and safety aspects of this program. Since this site remedial action is being undertaken pursuant to CERCLA, the requirements of 29 CFR 1910.120 apply. Therefore, to ensure a "qualified" person is responsible for health and safety, the contractor will be required to utilize the services of an Industrial Hygienist certified in Comprehensive Practice by the American Board of Industrial Hygiene. The CIH will be required to:

- posses a minimum of 3 years experience in developing and implementing health and safety programs at hazardous waste sites or in the chemical industry,
- have demonstrable experience in supervising professional and technician level personnel, and
- have demonstrable experience in developing worker exposure assessment programs and ambient air monitoring programs.

The CIH will have the primary responsibility for implementation, oversight, and enforcement of the health and safety aspects of this remedial action. It will not be necessary for the CIH to be on-site for the entire duration of field work. A fully trained and experienced Site Safety and health Officer (SSHO), responsible to the Contractor and the CIH, may be delegated to implement and continually enforce the safety and health program and site-specific plan elements on-site. The SSHO will be required to posses:

- a minimum of 1 year experience in developing and implementing health and safety programs at hazardous waste sites or in the chemical industry,
- demonstrated experience in construction safety techniques and procedures,
- a working knowledge of Federal and state health and safety regulations, and
- specific training in personal and respiratory protective equipment program implementation and in the proper use of air monitoring instruments, air sampling methods, and procedures.

Each crew actively working in the contaminated areas will be required to include a fully trained and experienced Safety and Health Technician to perform monitoring and ensure compliance with the approved SSHP.

The Contractor will be required to have at least one person certified in first air/CPR by the Red Cross, or equivalent agency, on-site during all site operations.

9.5 Training. All employees working on-site with the potential for exposure to hazardous substances, health hazards, or safety hazards shall meet the minimum training requirements as specified in 29 CFR 1910.120. These employees will have completed the 40 hour hazardous waste training requirements and have three days of field experience in hazardous waste work. All supervisory personnel will have an additional 8 hours of training as specified for management of personnel and activities associated with hazardous waste site activities. Documentation of this training will be required for all personnel; in addition documentation pertinent to annual refresher courses as required in 29 CFR 1910.120 will also be required. All employees will be required to attend site-specific training covering site hazards, procedures, and all contents of the approved SSHP prior to entering the site.

9.6 Personal protective equipment (PPE). Because of the nature of this work, it is likely that engineering controls and work practices will not provide sufficient control of the hazards, therefore, the contractor will be required to provide personal protective equipment to all affected employees. This PPE shall provide dermal and respiratory protection specific to the site hazards. Selection of appropriate PPE will be based on air monitoring results (for respiratory protection) and an evaluation of the potential for dermal exposure during each task (for dermal protection). The Contractor will be required to establish a written personal protective equipment program in compliance with 29 CFR 1910.120(g)(5). Basic levels of protection will be similar to those listed below. Historical information and past field activities in the Lime and M-1 Settling Basins have indicated the possible presence of chemical agents and their breakdown products. Therefore, the level of PPE required during intrusive activities shall be Level B.

9.6.1 Level D Protection:

- Hard hat
- Safety glasses with side shields or safety goggles.
- Work clothing as prescribed by weather.
- Steel toe work boots.
- Hearing protection (if needed)

9.6.2 Modified Level D Protection (all elements of Level D above plus:)

- disposable outer coveralls (tyvek or equivalent)
- disposable boot covers.
- Surgical inner gloves.
- Chemically protective outer gloves (as per PPE program).

9.6.3 Level C Protection:

- Hard hat
- Work clothing as prescribed by weather.
- Disposable outer coveralls (saranex coated tyvek or equivalent)
- Disposable boot covers
- Steel toe work boots.
- Hearing protection (if needed)
- Surgical inner gloves.
- Chemically protective outer gloves (as per PPE program).
- Air purifying respirator (APR) with appropriate cartridges (selected as per respiratory protection program).

9.6.4 Level B Protection:

all elements of Level C except air supplied respirators will be substituted for air purifying respirators.

9.7 Medical Surveillance. The contractor will be required to institute a medical surveillance program meeting the minimum requirements established by 29 CFR 1910.120. In order to ensure adequate medical surveillance for the hazards at this site, the contractor will be required utilize the services of a licensed physician who is certified in Occupational Medicine by the American Board of Preventative Medicine, or who, by necessary training and experience, is Board-eligible. The Contractor will be required to provide the physician with a copy of the employees' job descriptions, the SSHP, 29 CFR 1910.120, and Section 5.0 of NIOSH publication 85-115.

9.8 Exposure monitoring/air sampling program (personal and environmental). Because of the potential for airborne contamination, the contractor will be required to conduct air monitoring/sampling in order to establish proper levels of respiratory protection. Background conditions will be established prior to the start of work. As a minimum, real-time air monitoring for organic vapors and dust will be necessary within all work areas of an intrusive nature. Monitoring for chemical agents and arsine may also be required. This monitoring will continue throughout the duration of the activity.

9.8.1 In addition to the real-time monitoring, the contractor will be responsible for ensuring compliance with all requirements of 29 CFR 1910.120(h).

9.9 Standard operating safety procedures, engineering controls and work practices. All pertinent procedures will be addressed and implemented as described in the Contractor's SSHP.

9.10 Site control measures. Because contamination exists at this site, the Contractor will be required to establish work zones and site control measures to prevent the spread of contamination.

9.11 Personal hygiene and decontamination. Whenever employees are potentially exposed to contamination, they will be required to undergo decontamination procedures. The contractor will be required to set forth appropriate decon procedures for each level of protective clothing worn on-site. A personnel decon facility with shower facilities will be required.

Details about the disposal of trash, contaminated disposable PPE and decon water will be included in the specifications.

9.12 Equipment decontamination facilities and procedures. The Contractor will be required to decontaminate all equipment that has come into contact with contamination. The Contractor will be required to establish an equipment decon pad in the CRZ.

9.13 Emergency equipment and first aid requirements. The Contractor will be required to have the following items immediately available for on-site use:

- 9.13.1 First aid equipment and supplies approved by the consulting physician.
- 9.13.2 Emergency eyewashes/showers meeting the standards of ANSI Z-358.1
- 9.13.3 Emergency respirators (worst-case appropriate).
- 9.13.4 Spill control materials and equipment.
- 9.13.5 Fire extinguishers.

9.14 Emergency response plan and contingency procedures (on-site and off-site). The Contractor will be required to prepare an Emergency Response Plan in compliance with 29 CFR 1910.120(l), which addresses the following elements, as a minimum:

- 9.14.1 Pre-emergency planning and procedures for reporting incidents to appropriate government agencies for potential chemical exposures, personal injuries, fires/explosions, environmental spills and releases.
- 9.14.2 Personnel roles, lines of authority, communications.
- 9.14.3 Posted instructions and a list of emergency contacts (physician, nearby medical facility, fire and police departments, ambulance service, federal/state/local environmental agencies, CIH, Contracting Officer).
- 9.14.4 Emergency recognition and prevention.
- 9.14.5 Site topography, layout, and prevailing weather conditions.
- 9.14.6 Criteria and procedures for site evacuation (emergency alerting procedures/employee alarm system, emergency PPE and equipment, safe distances, places of refuge, evacuation routes, site security and control).
- 9.14.7 Specific procedures for decontamination and medical treatment of injured personnel.
- 9.14.8 Route maps to nearest pre-notified medical facility.

9.14.9 Criteria for initiating community alert program, contacts, and responsibilities.

9.14.10 Procedures for critique of emergency responses and follow-up. The Contractor will also be required to ensure all emergency response procedures set forth by RMA are followed.

9.15 Heat/cold stress monitoring. Ambient weather conditions will dictate when heat and cold stress monitoring requirements are appropriate. Ambient temperature readings and the type of clothing worn will affect the type and extent of monitoring required. The contractor will be required to provide and implement protocols for heat and/or cold stress monitoring.

9.16 Sanitation. The Contractor will be required to provide, in the Support Zone, potable water and washing facilities consisting of hot and cold running water, towels and soap for men and women as necessary. (See also paragraph 6.11: Personal hygiene and decontamination.) At least 1 toilet, and if there are more than 20 employees, at least 1 toilet seat and 1 urinal per 40 workers will be required. A sanitary break and lunch area will be required in the Support Zone.

9.17 Logs, reports, and recordkeeping. Proper documentation will be an important part of the remedial action. The contractor will be required to keep the following records:

9.17.1 OSHA Records. Required OSHA records are listed in Table 6-1.

9.17.2 Daily log and safety inspection reports. The daily log and safety inspection report shall include practices and events that affect safety and health, safety and health discrepancies encountered and safety and health issues brought to the supervisor's attention. Each entry shall include:

9.17.2.1 Date

9.17.2.2 Work area

9.17.2.3 Employees present in work area

9.17.2.4 PPE and work equipment being used in each area.

9.17.2.5 Special health and safety issues and notes

9.17.2.6 Signature of preparer.

CUTOFF WALLS AND CAP FOR LIME
AND M-1 SETTLING BASINS

COST

<u>DESCRIPTION</u>	<u>COST</u>
Sheet Piling	\$ 346,000
Site Improvements	189,500
Demolition	73,000
Monitoring Wells/Piezometers	149,500
Slurry Trench	1,290,500
Collection Trench	361,000
Collection Manhole	23,000
Access Road	33,000
Lime Basin Cap	476,500
Electrical	29,500
Force Main	<u>23,000</u>
Subtotal	3,047,500
Cost Growth x 1.036	3,157,000
Contingencies (15%)	<u>474,000</u>
Subtotal	3,631,000
S&E (8%)	<u>290,000</u>
Subtotal	3,921,000
Engineering & Design (1%)	<u>39,000</u>
Subtotal	3,960,000
Quality Assurance (1%)	40,000
Waste Management	<u>100,000</u>
TOTAL	\$ 4,100,000

CUTOFF WALLS AND CAP FOR LIME
AND M-1 SETTLING BASINS

SCHEDULE

The schedule provided is for work addressed in this Implementation Document. This includes the entire Lime Settling Basins IRA and a portion of the work associated with the M-1 Settling Basins IRA. The dates provided for the Final Implementation Document and Start Construction are interim dates and are provided for your information. The Implementation Deadline for completion is given in accordance with Paragraph 22.13 of the Federal Facility Agreement.

Final Implementation Document
Start Construction
Finish Construction

1 April 1991
15 January 1992
15 January 1993

REPORT DOCUMENTATION PAGE

Form Approved
OMB No. 0704-0188

Public reporting burden for this collection of information is estimated to average 1 hour per response, including the time for reviewing instructions, searching existing data sources, gathering and maintaining the data needed, and completing and reviewing the collection of information. Send comments regarding this burden estimate or any other aspect of this collection of information, including suggestions for reducing this burden, to Washington Headquarters Services, Directorate for Information Operations and Reports, 1215 Jefferson Davis Highway, Suite 1204, Arlington, VA 22202-4302, and to the Office of Management and Budget, Paperwork Reduction Project (0704-0188), Washington, DC 20503.

1. AGENCY USE ONLY (Leave blank)			2. REPORT DATE 11/00/90	3. REPORT TYPE AND DATES COVERED	
4. TITLE AND SUBTITLE IMPLEMENTATION DOCUMENT, CUTOFF WALLS AND CAP FOR LIME AND M-1 SETTLING BASINS, ROCKY MOUNTAIN ARSENAL, COLORADO, DRAFT			5. FUNDING NUMBERS		
6. AUTHOR(S)					
7. PERFORMING ORGANIZATION NAME(S) AND ADDRESS(ES) ARMY CORPS OF ENGINEERS. OMAHA DISTRICT OMAHA, NE			8. PERFORMING ORGANIZATION REPORT NUMBER 90312R01		
9. SPONSORING/MONITORING AGENCY NAME(S) AND ADDRESS(ES) ROCKY MOUNTAIN ARSENAL (CO.). PMRMA COMMERCE CITY, CO			10. SPONSORING/MONITORING AGENCY REPORT NUMBER		
11. SUPPLEMENTARY NOTES					
12a. DISTRIBUTION/AVAILABILITY STATEMENT APPROVED FOR PUBLIC RELEASE; DISTRIBUTION IS UNLIMITED			12b. DISTRIBUTION CODE		
13. ABSTRACT (Maximum 200 words) <p>THE INTERIM RESPONSE ACTION FOR THE LIME SETTLING BASINS CONSISTS OF 1) RELOCATION OF THE SLUDGE, 2) CONSTRUCTION OF A 360 DEGREE SUBSURFACE BARRIER, AND 3) CONSTRUCTION OF A SOIL AND VEGETATIVE COVER. A SLURRY TRENCH BARRIER AND 18" CAP WILL BE INSTALLED.</p> <p>THE IRA FOR THE M-1 BASINS REQUIRES 1) A 360 DEGREE SUBSURFACE BARRIER AND 2) IN-SITU VITRIFICATION. THE BARRIER WILL BE SHEET PILING.</p> <p>INFORMATION IS INCLUDED ON THE FOLLOWING ELEMENTS OF THE DESIGN REQUIREMENT:</p> <ol style="list-style-type: none"> 1. GEOLOGY AND HYDROLOGY 2. CONTAMINATION 3. SLURRY TRENCH CUTOFF WALL FOR LIME BASINS 4. SHEET PILE CUTOFF WALL FOR M-1 BASINS 5. VEGETATIVE COVER FOR LIME BASINS 6. CIVIL - GRADING, PAVING, DRAINAGE 7. WATER SUPPLY AND WASTEWATER COLLECTION <p style="text-align: right;"><i>DTIC QUALITY INSPECTED 8</i></p>					
14. SUBJECT TERMS COST, SLURRY, SPECIFICATIONS, IRA L, HEALTH AND SAFETY			15. NUMBER OF PAGES		
			16. PRICE CODE		
17. SECURITY CLASSIFICATION OF REPORT UNCLASSIFIED	18. SECURITY CLASSIFICATION OF THIS PAGE	19. SECURITY CLASSIFICATION OF ABSTRACT	20. LIMITATION OF ABSTRACT		